

Expert System for Selecting Concrete Constituents

By J.R. Clifton and B.C. Oltikar

Synopsis: A large amount of specialized factual and heuristic knowledge on the relations between the design of concrete mixtures, including the constituents, and the durability of concrete has been gained through research and field experience. Effective dissemination of this knowledge should result in fewer incidents of premature deterioration of concrete. Expert systems appear to be an effective means for transferring the knowledge on the durability of concrete obtained through laboratory and field studies and experiences to engineers and designers responsible for the design, construction, and maintenance of concrete structures.

Durcon is a prototype expert system being developed to give recommendations on the selection of constituents for durable concrete. The purpose of developing Durcon is to demonstrate the application of expert systems to improve the process of selecting construction materials. Four major deterioration problems are covered by Durcon, freezing and thawing, corrosion of reinforcing steel, sulfate attack, and alkali-aggregate reactions. This report discusses the approach being followed and the progress being made in developing Durcon. In addition, model systems for recommendations for concrete exposed to corrosive environments and for preventing alkali-aggregate reactions are presented.

Keywords: aggregates; alkali-aggregate reactions; artificial intelligence; computer programs; concrete durability; corrosion; freeze-thaw durability; mix proportioning; sulfate attack.

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Dr. James R. Clifton is the Leader of the Inorganic Building Group at the National Bureau of Standards. He is chairman of ACI Committee 222, Corrosion of Metals in Concrete.

Dr. Bhalchandra C. Oltikar is an Associate Professor in the Mathematics Department at the University of Puerto Rico, Mayaguez, Puerto Rico.

INTRODUCTION

Although extensive research has been carried out on the deterioration of concrete, persistent problems still occur with the durability of concrete structures. For example, the corrosion of reinforcing steel in concrete is a widespread occurrence in concrete structures exposed to chloride ions from deicing salts or seawater. Extensive research on corrosion of steel reinforcement has been carried out which has greatly increased the understanding of the factors affecting the corrosion processes. Similarly, extensive research has been carried out on other major deterioration problems, such as the freeze-thaw durability of concrete, sulfate attack of concrete, and deleterious reactions between alkalis in concrete and reactive aggregates. If the results of these and future research are properly disseminated, then fewer incidents of premature deterioration in new concrete structures should be observed.

Expert systems, also known as knowledge-based systems, appear to be an effective means for transferring the knowledge on the durability of concrete gained through research and field experience to individuals responsible for the design, construction, and maintenance of concrete structures. Expert systems are considered to be computer programs that use knowledge and inference procedures to solve problems that are difficult enough to require significant human expertise for their solution (1). The knowledge of most current expert systems consist of facts and heuristics. Facts constitute information that is widely accepted to be valid and readily available (e.g., in textbooks or ASTM standards). Heuristics are rules-of-thumb or good judgement that an expert would use to solve a problem when well-understood theories or facts are not available (2).

PRINCIPLES OF EXPERT SYSTEMS

An overview of expert systems is presented in this section to introduce the concepts involved in the



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development of Durcon. More comprehensive views of expert systems are given in references 3, 4, and 5.

Features of Expert Systems

In the several definitions given for the term "expert system," the essential concept expressed is the use of the knowledge of experts to solve difficult problems. Feigenbaum (1), a pioneer in expert systems, gave the following definitions in which the main concepts are presented:

"An Expert System is an intelligent computer program that uses knowledge and inference procedures to solve problems that are difficult enough to require significant human expertise for their solution. The knowledge necessary to perform at such a level, plus the inference procedure used, can be thought of as a model of the expertise of the best practitioners in the field."

Architecture of an Expert System

An expert system is an interactive system consisting of three major components (Figure 1):

- (1) A knowledge base of facts and heuristics which can be applied to a specific case.
- (2) A global data base (working memory) which contains, in temporary storage, observations or evidence provided by the user about a specific case, and all derived information about the case.
- (3) A control system or inference engine which selects the appropriate knowledge rules and recommendations for the solution of the problem.

Other features usually present in expert systems are:

- * explanatory interface which permits the user to understand why certain questions are asked and gives a justification for specific conclusions or recommendations
- * transparency of the computer program to the user (i.e., the user does not become involved in programming).

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- * modification of the computer program can be accomplished without a comprehensive re-programming, e.g., portions of the knowledge base can be modified without the need for other restructuring.

Knowledge Base

The ability of an expert system to solve a problem (referred to as its "power") has been observed to increase with the extent of its domain (subject specific) knowledge. Undoubtedly the most demanding phase of developing an expert system is obtaining and representing relevant knowledge.

The domain knowledge of an expert system is often represented by production rules which have the general form:

If: Logical conditions are satisfied (antecedent)
Then: Take the indicated action (consequence)

These rules are further illustrated by the following rule taken from Durcon:

If: (1) Severe freeze-thaw conditions are anticipated and
(2) The nominal size of aggregate is 3/8 inch (9.5 mm.)
Then: The percent of entrained air should be 7.5

An expert system usually has many judgmental or empirical rules (heuristics) which lead to recommendations or conclusions which contain a certain level of uncertainty. In some expert systems, a numerical value (certainty factor) are given to each rule to indicate the degree of certainty. These certainty factors can be combined with each other to obtain an overall certainty factor for the final recommendation or conclusion. Certainty factors are measures of the confidence that can be placed in any given conclusion or recommendation. The present version of Durcon does not include certainty factors because in the approach taken to develop all rules are considered to have a certainty factor of unity.

Control System

The control system decides which production rules should be invoked and it must resolve any conflicts which may occur if several rules are satisfied. There are two main approaches for evaluating production rules: backward chaining and forward chaining (3,5).

In backward chaining the system has a set of initial goals, such as solutions to a problem. The rules are invoked in reverse order, with the right-hand sides of the rules stating possible goals, and the left-hand side of the rules being examined to determine which goals are satisfied i.e., consistent with the evidence. The goals are proven or disproven until the "most-correct" solution is obtained. Backward chaining is often used in diagnostic systems e.g., MYCIN (6).

In forward chaining the system does not start with any defined goals. Instead, the system starts with a set of evidence and proceeds to invoke the rules in a forward direction (antecedent first), continuing until no further production rules can be invoked. Forward chaining is often used in systems providing recommendations on design related problems.

APPROACH FOR DEVELOPING DURCON

The knowledge base for expert systems is usually created as the result of interrogating an expert or a small team of experts by the knowledge engineer. In the development of the knowledge base for Durcon, a different approach is being taken. The "Guide to Durable Concrete," prepared by the ACI® (American Concrete Institute) Committee 201 on Durable Concrete (7) was chosen to be the factual component of the knowledge base. This document was selected because the facts it contains should have widespread credibility among the concrete community. In the conceptual design of Durcon, four major durability problems covered by the Guide were considered, corrosion of reinforcing steel, freezing and thawing, sulfate attack, and reactions between the constituents of cement and aggregates. These four problems were selected because they comprise the most persistent causes of the premature deterioration of concrete. The major factors controlling the response of concrete to these deterioration processes and recommendations of measures to minimize their deleterious effects are given in the Guide. In addition to the ACI Guide, appropriate ASTM standards, guidelines by the U.S.

Army Corps of Engineers, and Navy construction documents have been reviewed.

The first step in representing the factual knowledge used in Durcon was the creation of tree (hierarchy) structures. The tree structure for the knowledge of the corrosion of reinforcing steel is illustrated in

Figure 2. Then the facts were expressed in the form of production rules.

The sequences of the stages in developing Durcon is indicated in Figure 3. A subcommittee has been established in ACI Committee 201 for the purpose of critiquing Durcon and recommending what factual and heuristic knowledge should be added. Further improvements in the knowledge undoubtedly will result from user's evaluation during problem solving sessions.

MODEL SYSTEMS

Model systems have been developed for the four durability problems being addressed by Durcon. These model systems are largely based on the information contained in The ACI Guide to Durable Concrete (7), and they will be revised according to the recommendations from the Committee members and comments by users. The specific user inputs and recommendations given by each model system are listed in Tables 1-4. Because Durcon is still being developed, the model systems are for demonstrative purposes only and they, undoubtedly, will undergo significant enhancements.

Features of Systems

The model expert systems use production rules as described in the Knowledge Base section. Their control systems support a forward-chaining procedure (3). The programs are written in PASCAL and operate on the IBM PC (personal computer) or compatible, using the IBM PC disk operating system (DOS). The freeze-thaw, corrosion, and sulfate durability systems are contained on one diskette and require 256 KB of memory to operate. The alkali-aggregate system is on a separate diskette and requires a memory of 512 KB to operate. In the next major revision of Durcon, it will be restructured using an expert system shell which probably will permit the four durability systems to be contained on a single diskette.

Examples of Sessions

A users session with the corrosion knowledge base is presented in Table 5 and a session with the alkali-aggregate knowledge base is presented in Table 6. Note that with both systems, a user can request an explanation of both questions and of recommendations. A model system using the freeze-thaw knowledge base was previously published (8).

APPLICATIONS OF EXPERT SYSTEMS

Durcon addresses the selection of constituents and the design of concrete mixtures. Diagnostics and maintenance of reinforced concrete structures are areas in which expert systems should provide significant benefits. Diagnostics would include field inspection and laboratory analyses of concrete, the determination of the extent of deterioration, and the identification of the causes of deterioration. A system on maintenance would provide recommendations on the type and extent of repairs needed.

ACKNOWLEDGEMENT

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REFERENCES

1. E. Feigenbaum, "Expert Systems in the 1980's, in Machine Intelligence", ed. A. Bond, Pergamon Press (1981).
2. J. Pearl, "Heuristics", Addison-Wesley Publishing Company (Reading Massachusetts, 1984).
3. S.M. Weiss and C.A. Kulikowski; "A Practical Guide to Designing Expert Systems", Rowman and Allanheld Publishers (Totowa, New Jersey, 1984).
4. F. Hayes-Roth D.A. Waterman, and D.B. Lenat, (eds), "Building Expert Systems", Addison-Wesley Publishing Co. Inc. (Reading, Massachusetts, 1983).
5. R. Forsyth, "Expert Systems: Principles and Case Studies", (ed.), Chapman and Hall (London, 1984).
6. E.H. Shortliffe, "Computer-Based Medical Consultation: MYCIN", American Elsevier (New York, 1976).

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7. "Guide to Durable Concrete ACI 201.2R-77", Journal of the American Concrete Institute (December, 1977).
8. J.R. Clifton, B.C. Oltikar and S.K. Johnson, "Development of DURCON, An Expert System for Durable Concrete: Part 1", NBSIR 85-3186 (National Bureau of Standards, 1985).

Table 1. Corrosion Resistance of Reinforced Concrete: Specific Case Information and Recommendations

<u>Specific Case Information^a</u>	<u>Recommendations^b</u>
1. Environment: indoor or outdoors	1. Concrete cover over reinforcement
2. Indoor environment: little or no exposure to moisture, occasional, or regular exposure to moisture	2. Concrete permeability
3. Outdoor environment: sea or saltwater, arid climate, or normal to high precipitation	3. Water to cement ratio
4. Sea or saltwater exposure: splash zone, or continuous submergence	4. Chloride ion content
5. Chloride ion exposure: mild, moderate, or severe	5. Concrete quality
6. Reinforcement type: steel bars or prestressed steel	

^a Information provided by user on specific case being considered.

^b Recommendations by Durcon based on specific case information.

Table 2. Freeze-Thaw Durability: Specific Case Information and Recommendations

<u>Specific Case Information^a</u>	<u>Recommendations^b</u>
1. Type of cement: portland or blended	1. Water to cement ratio
2. Type of concrete: normal weight or lightweight	2. Amount of entrained air
3. Concrete member: thin section or normal section	3. Compressive strength
4. Exposure conditions: dry, moderate, or severe	4. Acceptibility of aggregate
5. Type of aggregate: natural aggregate or lightweight	5. Applicable ASTM standards and test procedures
6. Aggregate size	
7. Selection of admixture	

^a Information provided by user on specific case being considered.

^b Recommendations by Durcon based on specific case information.

Table 3. Sulfate Resistance: Specific Case Information and Recommendations

<u>Specific Case Information^a</u>	<u>Recommendations^b</u>
1. Exposure conditions: sulfate in soil or water	1. Water to cement ratio
2. Severity of exposure: sulfate contents of soil or water	2. Type of either portland or blended cement
3. Type of cement: portland or blended	3. If fly ash added: type of fly ash, and amount of fly ash added
4. Will fly ash be added to concrete?	4. Compressive strength
5. Type of concrete: normal weight or lightweight	5. Applicable ASTM standards

^a Information provided by user on specific case being considered.

^b Recommendations by Durcon based on specific case information.

Table 4. Alkali-Aggregate Reactions: Specific Case Information and Recommendations

<u>Specific Case Information^a</u>	<u>Recommendations^b</u>
1. Selection of aggregate by user based on either service record or aggregate testing	1. Acceptability of aggregate
2. Applicability of service record to present case	2. If aggregate is potentially reactive then recommendations on acceptability of cement given
3. Type of aggregate: siliceous or calcium carbonate	3. If aggregate is reactive and cement not of low alkali type then recommendation on acceptability of pozzolan or slag given
4. Results of petrographic analysis of aggregate	
5. Results of expansion and chemical testing of aggregate	

^a Information provided by user on specific case being considered.

^b Recommendation by Durcon based on specific case information.

Table 5. User Session on Corrosion of Steel Reinforcement in Concrete

(The computer displays text which conveys information, then requires user input in response to instructions or questions)

This section essentially addresses corrosion of reinforced steel induced by chloride ion. Other factors inducing corrosion are briefly mentioned.

The rate of corrosion of reinforcing steel in concrete is largely controlled by the following factors:

- 1) Concentration of chloride ions
- 2) Permeability of concrete
- 3) Electrical resistivity of concrete
- 4) Moisture condition of concrete
- 5) Temperature

Please answer the questions that the system asks, as well as you can. Hit return when you are ready.

Do you wish a list of some factors in addition to chloride ions which can induce corrosion?

- 1) Yes
- 2) No

1 (User types 1 and then Return)

LIST

- (1) CO_2 which reacts with Ca(OH)_2 to form CaCO_3 thereby reducing the pH of concrete.
- (2) H_2S can induce stress corrosion.
- (3) Other corrosive ions include thiocyanate, bromide, fluoride, sulfite sulfate, and nitrate.
- (4) Stray currents, e.g. from cathodic protection devices and electrical rail systems.

(Information on the severity of exposure is then sought)

Table 5. Continued

What is the concrete environment?

- 1) Outdoor
- 2) Indoor
- 3) Please Explain the Question
- 4) Redo a Previous Answer

Please enter a number from 1 to 4

1 Return

What is the climate or exposure of the concrete?

- 1) Seawater
- 2) Arid
- 3) Inland exposure to wet environment
- 4) Please Explain the Question
- 5) Redo the Previous Answer

Please enter a number from 1 to 5

4 Return

- Explanation of Question -

I am trying to determine the amount of water to which the concrete will be exposed. "Seawater" means the concrete is very near or even in the sea or a brackish body of water. "Inland exposure to wet environment" denotes exposure to moisture in form of rain, ice, snow or ground water.

Please enter a number from 1 to 5

3 Return

What is the extent of the chloride exposure?

- 1) Severe
- 2) Moderate
- 3) Mild
- 4) Please Explain the Question
- 5) Redo a Previous Answer

Please enter a number from 1 to 5

4 Return

- Explanation of Question -

I am trying to determine the amount of chloride to which the concrete will be exposed. Bridges, roads, sidewalks, and parking garages in areas where deicing salts are frequently used are subject to severe exposure. These structures are subject to moderate exposure in cases where deicing salts are

Table 5. Continued

less frequently applied. Structures near salt water or in areas with brackish soil are subject to mild exposure.

Please enter a number from 1 to 5

1 Return

What type of reinforcement steel will be used?

- 1) Prestressed
- 2) Steel Reinforcement bars
- 3) Please Explain the Answer
- 4) Redo a Previous Answer

Please enter a number from 1 to 4

2 Return

*****RECOMMENDATIONS*****

(OUTPUT)

Good workmanship is extremely important. Low slump, air entrained and well graded aggregates should be used. Seven days of uninterrupted moist or membrane curing is recommended. To prevent stray currents, avoid contact between different metals in the concrete and be cautious of protruding metals. Good drainage is important. A 2" cover and a water/cement ratio below 0.40 or a 2.5" cover or and a water/cement ratio below 0.45 is necessary. See ACI 211.1-74 for cement content.

For bridge deck or similar structure exposed to severe chloride ion environment, I recommend that additional protection is provided to reinforcing steel by using either or combination of:

- (1) Low slump dense concrete (w/c of 0.32)
- (2) Epoxy coated reinforcing steel
- (3) Waterproof membranes.

Maximum chloride content should be 0.10%.

Table 6. User Session on Alkali-Aggregate Reactions

RECOMMENDATIONS FOR PREVENTING DELETERIOUS REACTIONS BETWEEN
ALKALIES AND AGGREGATES

This system starts with a discussion of alkali-aggregate reaction problems and methods to avoid the problems.

Are you interested in discussion?

- 1) Yes
- 2) No
- 1 Return

DESCRIPTION OF ALKALI-AGGREGATE REACTION PROBLEMS

All aggregate react to some degree with alkalis in cement, however, in the majority of the cases the reactions are beneficial or innocuous. In some cases, however, enough expansive products are formed to cause concrete cracking. The expansion is attributed to the imbibition of water by the reaction products. When cracking occurs its effects can range from being cosmetic to resulting in complete failure of the structure necessitating its replacement. Alkali-Aggregate reaction may proceed rapidly (several months to a few years) or much slower (for example, as slow as 30 years or more).

Expansive alkali-aggregate reactions are known to occur with certain siliceous aggregates and certain aggregates composed of carbonates (particularly dolomitic carbonates). Two categories of deleterious reactions between siliceous aggregates and alkalis have been described:

- 1) Alkali-Silica reaction and;
- 2) "Sand-gravel problem" or the "Cement-aggregate reaction"

The magnitude of the expansive reactions depend on:

- * The specific aggregate
- * Alkali content of the cement and concrete
- * Availability of extraneous alkalis (e.g., sea water)
- * Availability of water

METHODS OF AVOIDING PROBLEMS

Methods currently used to avoid deleterious consequences of alkali-aggregate reactions are:

- 1) Do not use reactive aggregates which produce deleterious expansions.
- 2) Use cement with low alkali contents.

Table 6. Continued

- 3) If reactive aggregate and high alkali cement must be used in combination use a mineral admixture (Pozzolan or Slag) to prevent deleterious alkali-aggregate reactions.

The preferred method is (1).

Why?

Method (1). If aggregate is "unreactive" then constraints are not imposed on the cement or the environment.

Method (2). In addition to alkalies of cements, alkalies can come from extraneous sources such as seawater. Also, low alkali cements may not be locally available.

Method (3). Depends on availability of pozzolan or slag. Mortar bar expansion testing lasting possibly as long as 6 months may be required.

(Query Session Starts)

Methods to be selected to avoid potential alkali-aggregate reaction problems

- 1) Use aggregate which does not produce disruptive expansion (i.e., cracking of concrete).
- 2) Use cement with low alkali content
- 3) Use pozzolan or slag
- 4) Please explain the question
- 1 Return

Is service record of acceptable aggregate performance (i.e., record of performance in a structure) available?

- 1) Yes
- 2) No
- 3) Do not know
- 4) Please Explain the Question
- 3 Return

- Explanation of Question -

In most cases aggregates are selected on bases of known satisfactory service record. This is usually sufficient if aggregate's mineralogy, the cement, the concrete and the exposure are the same as those on which service record is based.

Please type your answer

- 1 Return

Table 6. Continued

The following questions are intended to assist you in determining if the past service record can be used in determining if aggregate will be reactive in the present case.

Any changes in quarry which could affect aggregate mineralogy?

- 1) Yes
- 2) No
- 3) Please Explain the Question
- 2 Return

Is there an increase in alkali content of cement or concrete on which service record is based?

- 1) Yes
- 2) No
- 3) Please Explain the Question
- 2 Return

Same exposure condition as in service records?

- 1) Yes
- 2) No
- 3) Please Explain the Question
- 2 Return

*** Recommendation ***

Because of the changes in exposure we recommend that aggregate be tested for susceptibility to alkali-aggregate reactions.

Do you plan to determine the aggregate reactivity?

- 1) Yes
- 2) No
- 1 Return

The following will give RECOMMENDATIONS for determining REACTIVITY OF AGGREGATE

Select the Type of Aggregate

- 1) Siliceous
 - 2) Carbonate
 - 3) Don't know
 - 4) Please Explain the Question
 - 1 Return
-

Table 6. Continued

The following will give RECOMMENDATIONS for determining
REACTIVITY OF AGGREGATE

Select the Type of Aggregate

- 1) Siliceous
- 2) Carbonate
- 3) Don't know
- 4) Please Explain the Question
- 1 Return

Do you desire a compilation of reactive forms of silica in rock
and known rocks containing reactive silica?

- 1) Yes
- 2) No
- 2 Return

Recommend procedure is petrographic examination (ASTM C-295)
followed by expansion test (ASTM C-227) and chemical test
(ASTM C-289).

Do you wish to see the reason for recommendation?

- 1) Yes
- 2) No
- 2 Return

If petrographic examination previously performed, or not possible
to perform then proceed directly to Expansion and chemical test.

Which of the following?

- 1) Petrographic examination followed by expansion and chemical
tests
- 2) Expansion and Chemical Tests
- 1 Return

Does examination by the experienced petrographer using ASTM
method C-295 disclose any of the following?

- * Opal
- * greater than 5% chert in any chalcedony
- * greater than 1% tridymite or cristobalite or quartz
- * greater than 20% strained quartz

- 1) Yes
- 2) No
- 2 Return

Table 6. Continued

Does aggregate contain MORE THAN 20% strained quartz?

- 1) Yes
- 2) No
- 3) Please explain the question
- 2 Return

Is aggregate classified by the petrographer as potentially causing "Sand Gravel problem"?

- 1) Yes
- 2) No
- 3) Information on "Sand Gravel problem"
- 2 Return

**** Recommendation ****

Expansion and chemical tests are recommended in addition to petrographic examination.

Do you want to see the reason for recommendation?

- 1) Yes
- 2) No
- 1 Return

Reason: Reactive silica in amounts too small to be detected by petrographic examination may be present which may produce sufficient expansion to cause concrete deterioration.

Did petrographic examination disclose presence of mineral which cause anomalous results with ASTM C-289 (Chemical method)?

- 1) Yes
- 2) No
- 3) Please explain the question.
- 2 Return

Is a test period of at least 3 months permitted?

- 1) Yes
- 2) No
- 3) Why
- 1 Return

Perform both ASTM C-227 (Expansion Test) and C-289 (Chemical Test).

Table 6. Continued

Does the aggregate pass both ASTM C-227 and C-289?

- 1) Yes
- 2) No
- 3) Marginal answer with C-227
- 1 Return

RECOMMENDATION

Both petrographic examination and laboratory tests indicate that the aggregate is sufficiently nonreactive to be used.

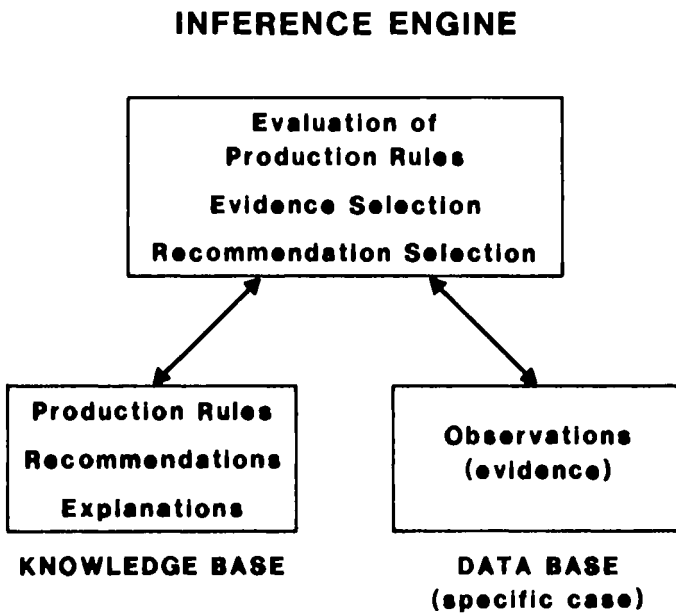


Fig. 1--Major components of an expert system

CORROSION RESISTANCE OF REINFORCED CONCRETE

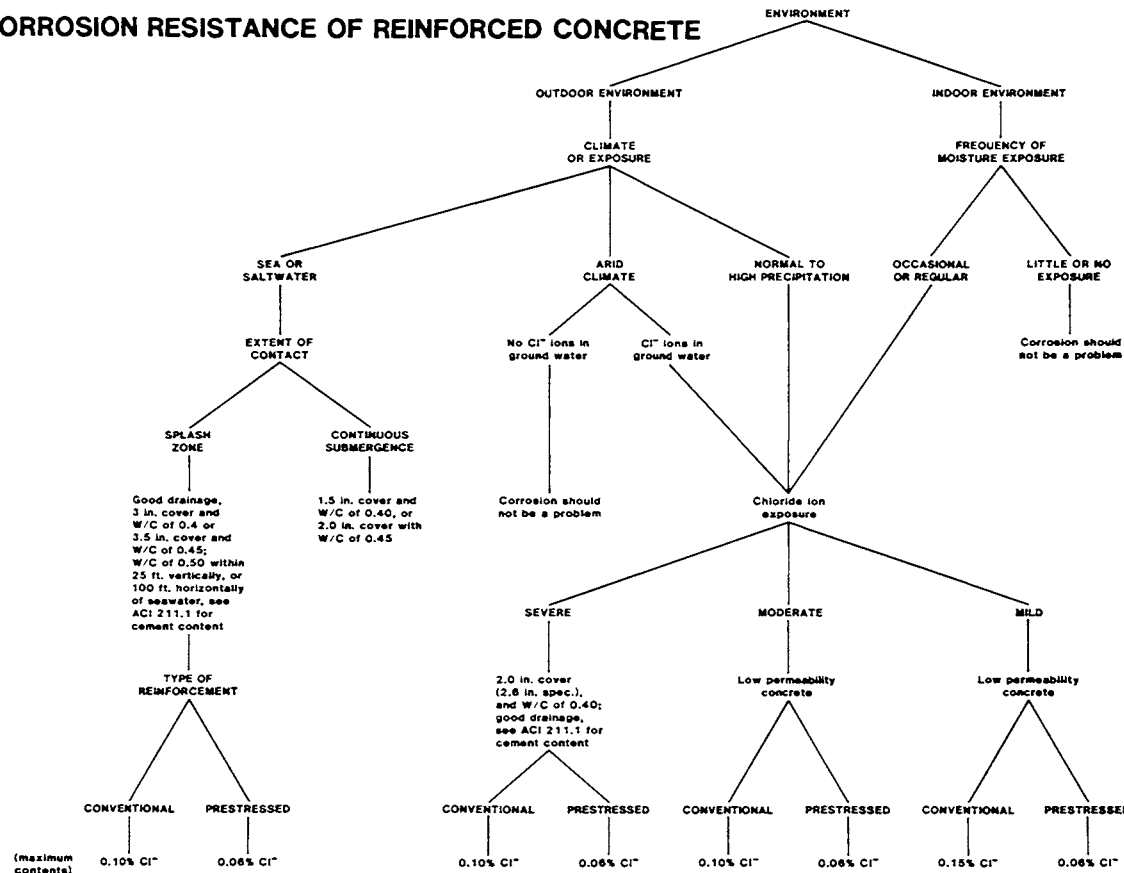


Fig. 2--Tree structures for representing knowledge of corrosion of reinforcing steel

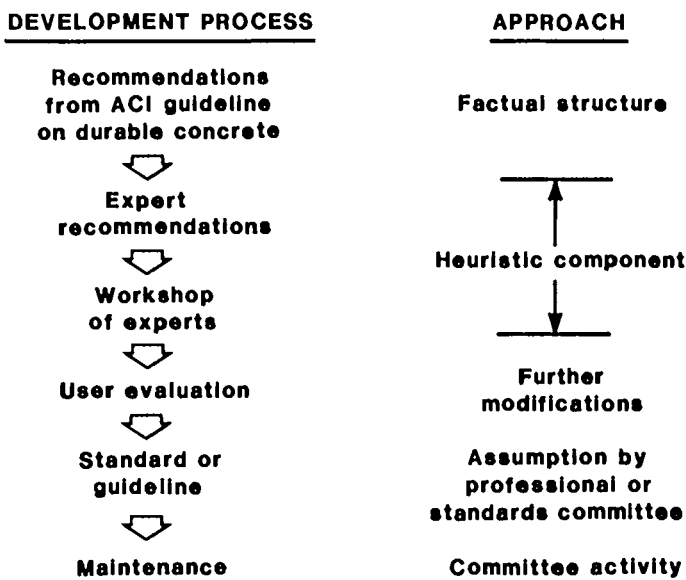


Fig. 3--Sequence of the development of Durcon

Nonlinear Analysis of Reinforced Concrete Shells

By A.C. Scordelis and E.C. Chan

Synopsis: With the aid of modern digital computers and sophisticated computational techniques such as the finite element method, it is now possible to simulate the structural behavior of an arbitrary reinforced concrete shell structure under general loading through its elastic, cracking, inelastic, and ultimate load ranges, taking into account nonlinear material, nonlinear geometry and time-dependent effects of creep and shrinkage.

In this paper, a method of analysis and a computer program based upon a composite layered finite element displacement model are briefly described. The analysis recognizes the nonlinearities due to cracking, nonlinear stress-strain behavior in concrete, yielding of the steel reinforcement and the tension stiffening between cracks. The effects of the continuously changing structural geometry are taken into account by an updated Lagrangian formulation. The time dependent effects of creep and shrinkage are also included by an initial strain procedure. Numerical results for reinforced concrete shells, obtained with the computer program are presented which indicate that in some cases an increase and in other cases a large reduction in the calculated ultimate load occurs as each of the nonlinear factors is included in the computer analysis.

Keywords: computer programs; cracking (fracturing); creep properties; edge beams; elastic properties; finite element method; reinforced concrete; shells (structural forms); shrinkage; strength; structural analysis.

Alex C. Scordelis is professor of civil engineering at the University of California, Berkeley, where he has been a member of the faculty since 1949. Noted for his teaching, research, and consulting in the fields of structural theory, reinforced and prestressed concrete, and shell and bridge structures, he is a member of ACI Committee 334, Concrete Shell Design and Construction, and Committee 343, Concrete Bridge Design, and since 1978, of the National Academy of Engineering.

Esmond C. Chan is a Senior Engineer with the Structural Software Development Co., Berkeley, California, where he is principally involved in the analysis and design of arctic offshore structures and pipelines. He received his Ph.D. in 1982 from the University of California at Berkeley, where he did extensive research in the computer analysis of concrete shell and bridge structures.

INTRODUCTION

General Remarks

A reinforced concrete shell has the qualities of an ideal structure. From an economical viewpoint, it requires the minimum material to cover a given area. From an architectural viewpoint, the structure is aesthetically appealing. From an engineering viewpoint, the structure is very strong and has a tremendous load-carrying capacity. It is for these reasons that reinforced concrete shells have found applications for the roofs of sport arenas, exhibition halls, and other long-span roof structures.

In the past, their design was usually based upon membrane theory and, to some extent, classical shell theory, assuming a linear, homogeneous, uncracked material. The continuous demand for longer spans and thinner shells, the failure of some shells several years after their construction, and the large deflections observed in other shells underline the importance of using realistic and rational analytical methods to ensure the safety and the serviceability of these shells throughout their useful life.

Recognizing this need, a continuing program of research on linear and nonlinear analysis methods and computer programs for reinforced concrete shells has been conducted at the University of California at Berkeley, since 1960.

Purpose of Paper

In this paper, a brief summary of a large number of linear and nonlinear analysis computer programs for shells and folded plates developed at Berkeley will be presented. For easy

reference the acronym names of the programs are cited in the text of the paper in parentheses and their full titles and other reference numbers are given in Appendix A. Details of the input-output capabilities and the availability of any of these programs can be obtained from the senior author.

However, the main purpose of this paper is to describe in some detail the most recent nonlinear analysis computer program (NASHL) developed at Berkeley, which is based upon a composite layered finite element displacement model. The analysis recognizes the nonlinearities due to cracking, nonlinear stress-strain behavior in concrete, yielding of the steel reinforcement and the tension stiffening between cracks. The effects of the continuously changing structural geometry are taken into account by an updated Lagrangian formulation. The time dependent effects of creep and shrinkage are also included by an initial strain procedure.

Numerical results for reinforced concrete shells, obtained with the computer program NASHL, are presented to indicate the effects on the structural behavior and the ultimate load capacity due to: (1) cracking, (2) nonlinear material, (3) time-dependent effects of creep and shrinkage, and (4) nonlinear geometry. It was found that in some cases an increase and in other cases a large reduction in the calculated ultimate load occurs as each of these nonlinear factors is included in the computer analysis. It is important that future designs of long span reinforced concrete shells be checked with computer programs which can account for the factors described above.

LINEAR ANALYSIS PROGRAMS

At Berkeley, during the past 25 years many linear analysis programs have been developed in which the shell is assumed to be made of a linear, homogeneous, uncracked material.

The methods of analysis used in the various programs have been: (a) the classical membrane or bending theory; (b) the finite element method; and (c) the finite strip method.

Special purpose computer programs for the following shell types are available: (1) folded plates; (2) cylindrical shells; (3) domes and cylinders; (4) hyperbolic paraboloids; (5) HP, elliptical and cylindrical groin vaults; and (6) shells of arbitrary geometry. Whenever possible, the important shell-edge member interactions have been incorporated into the programs and all of the internal forces, moments and displacements needed in the detailed design of the structure are output.

Folded Plates and Cylindrical Shells

Initially, programs for typical prismatic folded plates, simply supported at the two ends were written using ordinary or

elasticity theory, together with a direct stiffness harmonic analysis (FOLPLOR, FOLPLEL).

These programs were later extended to more general loadings (MULTPL) and to prismatic shells made up of any general assembly of cylindrical shell, folded plate and edge beam elements (MULEL, MULELR).

Then by using a force method in combination with the classical direct stiffness harmonic analysis, program capabilities were successively added so that continuous prismatic shells made up of these elements and supported by interior planar frames could be analyzed (MUPDI, MULDI, MUPDI3, MUPDI4).

In order to treat simply supported prismatic shells, straight or circularly curved in plan and made up of orthotropic instead of isotropic elements, programs similar to the above were written using the finite strip method (CURSTR, CURDI, CURDI4).

Finally in order to treat general end and interior support conditions for prismatic and non-prismatic folded plate systems, special purpose finite element programs were written (FINPLA, CELL, FINPLA2, CELL4).

Domes, HP Shells, and Ground Vaults

In the mid 1960's the research effort was shifted to the development of computer programs for the linear analysis of spherical domes, translational or other double curvature domes, hyperbolic paraboloids and groined vaults.

For spherical domes, programs were developed using classical membrane theory plus the Geckeler approximation for bending effects which made it possible to analyze a dome alone, a dome-ring beam system, or a dome-ring beam-cylinder system (DOME, DOMCYL). For general axisymmetric geometry shell-edge beam systems, an axisymmetric finite element program (AXISB) was developed, which proved to be quite useful.

A series of programs were also written using classical membrane analysis for HP shells made up of parallelograms (HYPARP) or arbitrary quadrilaterals (HYPARQ) in plan as well as for hyperbolic paraboloid, elliptical paraboloid, or cylindrical groined vaults (GROINV, LSGNV). These membrane analysis programs require a minimum of input and computer time and can be used for preliminary analysis. For final analysis and design a finite element program (NASHL) which takes into account membrane and bending effects and edge beams interactions can be used.

For the linear or nonlinear analysis of shells of arbitrary geometry, with or without edge beams, general finite element programs such as the ones to be described in the next section have been developed (NARCS, NOTACS, NOPARC, NASHL).

NONLINEAR ANALYSIS PROGRAMS

About 13 years ago (1973), research on the nonlinear analysis of reinforced concrete shell structures was begun at Berkeley, recognizing that the demand for longer spans and thinner shells made it desirable to have refined analytical models and computer programs, which could trace the complete structural response of these structures throughout their service load history and under increasing loads up to ultimate failure.

The capabilities desired were the nonlinear material, geometric and time dependent analysis of reinforced concrete shells with edge beams through their elastic, cracking, inelastic and ultimate ranges. The time dependent effects of load and temperature history and creep and shrinkage of the concrete should be taken into account.

Initially a computer program (NARCS) which incorporated only the nonlinear material properties of the concrete and reinforcing steel was developed, and then successively at about three year intervals time-dependent effects (NOTACS), then edge beams and then nonlinear geometry (NOPARC, NASHL) were added. The present computer program for reinforced concrete shells with edge beams (NASHL) incorporates all of these effects and gives as final output for any load and time up to failure: (1) joint displacements; (2) internal strains and stresses in the concrete and reinforcing steel; and (3) crack patterns.

A detailed description of the theoretical development, analytical models and elements, material properties, solution techniques, and input-output for the computer program NASHL can be found in Reference A 14. A brief description of some of these features is given below.

Analytical Model

In the computer program, the analytical model consists of a series of joints interconnected by shell and beam elements (Fig. 1). The shell element is a nine node, two dimensional curved isoparametric element with 5 DOF at each node (Fig. 2). The element thickness is divided into concrete and reinforcing steel layers and each layer is assumed to be under a two dimensional stress state. The steel reinforcement can be placed in any layer and in several directions if desired. Cracking and nonlinear material response is traced layer by layer under increasing load and the time dependent effects of creep and shrinkage. An updated Lagrangian formulation is used to take the effects of changing structural geometry into account.

By adding two straight one dimensional beam elements to the side of a curved shell element, thin shells with edge beams can be analyzed (Fig. 3). Each beam element with 12 DOF is prismatic (Fig. 4); but has an arbitrary cross-section made up of discrete

numbers of concrete and reinforcing steel filaments, instead of layers, for which the uniaxial strains and stresses are monitored.

In order to simulate elastic supports, obtain support reactions, or enforce irregular boundary conditions, linear elastic spring elements can be placed in any specified direction at discrete nodal points.

Concrete Material Properties

For the concrete layers of the shell elements, a biaxial orthotropic material law based upon nonlinear elasticity, together with a failure envelope, are used to represent the nonlinear behavior of the concrete using equivalent uniaxial strains. A typical equivalent uniaxial stress-strain curve, which is dependent on the existing biaxial stress ratio, consists in compression of a parabolic curve up to the point of maximum stress ($\sigma_{ic}, \epsilon_{ic}$), followed by a linear unloading curve to the point of final concrete crushing ($\sigma_{cu}, \epsilon_{cu}$), where $\sigma_{cu} = 0.2 \sigma_{ic}$ and $\epsilon_{cu} = 4 \epsilon_{ic}$. In tension, smeared cracking over each shell element layer at a limiting tensile stress σ_{it} and two directional cracks orthogonal to each other are assumed. Once cracking occurs, in one direction, the concrete stress normal to the crack drops to zero and the concrete behaves as a uniaxial material in the orthogonal direction until both directions crack. At this point, the concrete can only carry shear stress by shear interlock between cracks. This is done by modifying the shear stiffness by a shear retention factor β , where $0 < \beta < 1$.

Reinforcing Steel Properties

A bilinear uniaxial stress-strain relationship is assumed for the steel. Steel yields with a reduced modulus E_{sh} after reaching its yield strength f_y and any subsequent unloading is assumed to occur with the initial modulus E_s . The steel fails at the limiting strain of ϵ_{su} .

Once concrete cracks, the stress normal to the crack is zero; however, the presence of the steel reinforcement will permit concrete to carry some tension between cracks. This additional tension is conveniently lumped at the steel level and appears as an added tension stiffening to the steel. This procedure is used in NASHL because it takes into account the effects of the steel location in the thickness direction and the crack direction relative to the reinforcing direction.

Torque-Twist Relationship for Beam Elements

A trilinear model is used for the effective torque-twist response for the reinforced concrete beam elements. The curve is completely defined by three points: the torsional moment T_{cr} at

cracking and the corresponding twist α_{cr} , the torsional moment T_{yp} at full yielding of the longitudinal steel reinforcement and the corresponding twist α_{yp} , and the ultimate twist at failure α_u . These points can be found from available experimental test data. In the present NASHL program it is assumed that longitudinal bending and torsional effects in the beam elements are completely uncoupled.

Creep and Shrinkage of the Concrete

A numerical formulation, developed by Kabir[A12] is used for creep of the concrete in which the principle of superposition is assumed to be valid for evaluating the creep strain $\epsilon^c(t)$ at any time t as expressed by the following superposition integral:

$$\epsilon^c(t) = \int_0^t C(\tau, t - \tau, T) \frac{\partial \sigma(\tau)}{\partial \tau} d\tau \quad (1)$$

in which $C(\tau, t - \tau, T)$ is the specific creep function dependent on age of loading τ and temperature variation T , and $\sigma(\tau)$ is the stress applied at time τ .

The total creep strain at any time t can be found as the sum of the independent creep strains produced by the stress changes at different ages with different durations of time up to t .

For the effect of temperature variations on creep, the concrete is assumed to obey the time shift principle.

Stress and temperature changes are assumed to occur only at distinct time steps t_n ; $n=1, 2, \dots, N$. And, for the calculation of the creep strain increment during a given time interval, the stress and the temperature are assumed to remain constant.

It has been found that certain forms of mathematical approximations for the specific creep function, while representing experimental or empirical creep curves accurately, overcome the necessity of storing all the stress increments of previous time steps. Such a form is adopted as follows:

$$C(\tau, t - \tau, T) = \sum_{i=1}^m a_i(\tau) [1 - e^{-\lambda_i \phi(T)(t-\tau)}] \quad (2)$$

in which m , $a_i(\tau)$, λ_i , $\phi(T)$ are determined by a least squares fit to experimental or empirical creep curves, such as those recommended by ACI or CEB-FIP.

An efficient numerical procedure for evaluating creep strains can be developed using the following definitions for incremental quantities of time steps, stresses, and creep strains:

$$\Delta t_n = t_n - t_{n-1} \quad (3)$$

$$\Delta \sigma_n = \sigma_n - \sigma_{n-1} \equiv \sigma(t_n) - \sigma(t_{n-1}) \quad (4)$$

$$\Delta \epsilon_n^c = \epsilon_n^c - \epsilon_{n-1}^c \equiv \epsilon^c(t_n) - \epsilon^c(t_{n-1}) \quad (5)$$

Combining Eq. (1) to (5), after some extensive algebraic manipulations, the recursive relations necessary for calculating the increment of creep strain $\Delta \epsilon_n^c$ at a time step t_n are as follows:

$$\Delta \epsilon_n^c = \sum_{i=1}^m A_{i,n} [1 - e^{-\lambda_i \phi(T_{n-1}) \Delta t_n}] \quad (6)$$

$$A_{i,n} = A_{i,n-1} [e^{-\lambda_i \phi(T_{n-2}) \Delta t_{n-1}}] + \Delta \sigma_{n-1} a_i(t_{n-1}) \quad (7)$$

$$A_{i,2} = \Delta \sigma_1 a_i(t_1) \quad (8)$$

A very important advantage of the above formulation is that the computation for each new creep strain increment requires only the stress history of the last time step and not the total stress history. The above procedure is applied in a uniaxial direction in the filaments of the beam elements and in the biaxial principal stress directions in the layers of the shell elements.

The shrinkage strains can be found from experimental data or empirical formulas such as those recommended by ACI or CEB-FIP. These shrinkage strains are non-stress originated and are assumed to be composed of axial components only and are uniform in all directions.

Nonlinear Geometry Effects

The important steps in handling nonlinear geometry are the inclusion of the strains due to rotations and the updating of the structural geometry.

The procedure used in this study is based upon small strains and small incremental rigid body rotations. An updated Lagrangian formulation is used where all the stress and strain states are referred to the last known configuration. The assumption of small strains implies linear superposition of strain increments and the assumption of small rotations implies that rotations can be treated as vectors.

Methods of Analysis

The external loads R are assumed to be applied only at the joints (Fig. 1). The external joint load history, temperature history, short-time stress-strain curves, time-dependent material properties, and boundary conditions are given. The unknown joint displacements r and the internal strains ϵ and stresses σ in the concrete and the reinforcing steel, at any point in the structure are to be found for any instant of time. The resulting load-displacement (R vs. r) relationship will be nonlinear because of possible nonlinear material, geometric, or time-dependent effects.

To incorporate time-dependent nonlinearities, the time domain is divided into a discrete number of intervals, and a step forward integration is performed in which increments of displacements and strains are successively added to the previous totals as the solution marches forward in the time domain.

At each step, a direct stiffness analysis based on a displacement formulation is performed in the space domain, in which the resulting equilibrium equations will be nonlinear to be valid for the current state of material properties and geometry.

To account for geometric nonlinearity, an updated Lagrangian formulation is used, in which the direction of the local coordinate system is continuously updated as the structure deforms. Internal forces and stiffnesses are calculated in the local coordinate system for each element and transformed to the fixed global coordinate system where the equilibrium equations for the entire structure are assembled by the direct stiffness method and solved. Thus, the continuously changing displacement transformation matrix for each element takes into account the effect of geometric nonlinearity along with the nonlinear form of the strain-displacement relationship.

The total, (Eq. 9a) or tangential, (Eq. 9b), equilibrium equations in matrix form are:

$$K_r = R \quad (9a)$$

$$K_t dr = dR \quad (9b)$$

in which the stiffness matrices K or K_t are functions of displacement r and material properties. Eqs. (9a) or (9b) are solved in the computer using an incremental load method with iterations within each load increment.

In NASHL an option is provided to use either the tangent stiffness or a constant stiffness during each single or series of iterations. In addition, a controlled displacement procedure can be specified when structures showing strain softening or snap through phenomena are to be analyzed.

The NASHL computer program makes it possible to analyze reinforced concrete shells of arbitrary shape with edge beams. At each time step external gravity, surface, pressure and joint load increments as well as displacement increments can be specified as input. Several types of analysis can be performed: (1) linear elastic analysis; (2) nonlinear analysis with any of following -- nonlinear geometry, nonlinear material, time dependent effects -- as specified in the input.

Numerical results from two examples obtained using NASHL are presented below to illustrate the applications and capabilities of the program. Example 1 is a cylindrical shell with edge beams and Example 2 is a gabled HP shell with crown and edge beams.

EXAMPLE 1 - CYLINDRICAL SHELL

A series of eleven cylindrical shells, simply supported at the two ends and constructed of reinforced concrete, have been tested experimentally by Bouma et al [A14]. The shells were 1/8 scaled models of actual full scale structures, and they had identical cross sections and edge beams, but different span lengths and amounts of reinforcement. In this example, the shell designated A2 in the test series is chosen (Fig. 5). The loadings on this model were the shell load of 41 psf of surface and a load of 33.6 lb/ft on each edge beam, and an additional 25% of these loads to account for the live load. The total load on the model was therefore 3360 lb. In the test, this load was proportionally increased until failure occurred.

A linear analysis of the unreinforced shell is first performed using NASHL to test the finite element model. Then nonlinear analyses are carried out to determine the nonlinear response and the ultimate load capacity of the shell.

Linear Analysis

The geometry of the shell is shown in Fig. 5. Due to symmetry, one-quarter of the shell is modelled by 6 shell elements and 6 beam elements (Fig. 6). The shell element is divided into 6 layers through the thickness while 10 layers are used for each of the two directions of the beam's cross section.

The loading on the model is scaled proportionally to obtain a reference load of 1.0 psi on the shell's surface and 7.78 lb/in on each edge beam. The distributed load is replaced by the tributary nodal loads in the analysis.

The material properties of the shell are assumed to be:

$$E = 4.35 \times 10^3 \text{ ksi}$$

$$\nu = 0.3$$

The results are compared with those obtained from the computer program MULEL [B7], which is based upon an analytical solution using classical shell theory for the shell and ordinary beam theory for the edge beam. Comparisons are presented in Figs. 7 and 8 for the displacements and the internal forces and moments in the shell. It can be seen that good agreement exists between the results of the present analysis by NASHL and those from MULEL, even for the relatively coarse mesh used in the present study.

Nonlinear Analysis

The reinforced concrete shell is first analyzed including nonlinear material alone and, secondly, including both material and geometric nonlinearities.

The same mesh layout used in the linear analysis is again used. The reinforcement details used in the analysis are shown in Fig. 9. The total steel area in the edge beam is replaced by 10 equally spaced steel filaments.

The material data for the concrete and steel show a substantial scattering in the tests and thus the following mean values are used in the analysis:

<u>Concrete</u>	<u>Steel</u>
$E_o = 4.37 \times 10^3 \text{ ksi}$	$E_s = 29.9 \times 10^3 \text{ ksi}$
$f'_c = 4.12 \text{ ksi}$	$E_{sh} = 0.0$
$f'_t = 0.711 \text{ ksi}$	$f_y = 42.7 \text{ ksi}$
$\nu = 0.3 \text{ (assumed)}$	$\epsilon_{su} = 0.1$
$\epsilon_c = 2 f'_c / E_o$	

The resulting load-displacement response for the shell, obtained using NASHL in the present study, is presented in Fig. 11, together with test results of Bouma et al. The agreement between the analytical and test results is satisfactory for the case where nonlinear geometry is included.

The distinct increase in stiffness due to the increase in the T-C couple when nonlinear geometry is included is particularly noted. For the analysis including nonlinear material only, an ultimate load of 8.4 k is obtained. This is close to the value obtained from the beam theory. However, when nonlinear geometry is included in the analysis, an ultimate load of 10.2 k is obtained. This represents a 21% increase in strength over the case where nonlinear geometry is not considered.

The experimental ultimate load reported was 11.8 k. The ultimate load obtained in the present analysis is therefore 14% lower than the experimental value.

Investigating the mode of failure for the case where nonlinear geometry is included indicates that the shell sustained initial yielding of the longitudinal steel causing larger displacements and more cracking and then finally failed in diagonal tension near the supporting points. The crack patterns are shown in Fig. 10.

This failure mode suggests that a more refined steel stress-strain relationship has to be used in order to obtain a better correlation in the ultimate load range. The modelling of the steel reinforcement as an elasto-plastic material is insufficient in this case.

A plot of the total load vs. the tensile forces at mid-span is shown in Fig. 12. The smaller tensile forces for the same external load is noted for the analysis with nonlinear material and geometry. This decrease in the tensile forces is due to the increase in the moment arm due to the change in geometry of the structure.

At a total load of 8.4 k, which corresponds to the ultimate load of the shell when nonlinear geometry is excluded, the tensile force, T_1 , at mid-span is 14.8 k. The moment arm, d , can be found by assuming T_1 to be at the centroid of the edge beam: $d = (M_e)/(T_1) = 9.38$ in.; where M_e is the external moment in equilibrium with the T-C couple produced by T_1 .

When the change in geometry is considered, an increase in moment arm of $\Delta = 0.76$ in. is obtained. The tensile force in this case is decreased to $T_2 = (M_e)/(d+\Delta) = 13.7$ k; which is in good agreement with the analysis including nonlinear geometry which gives a tensile force of 13.5 k.

A statical check has also been carried out at mid-span, for the case where nonlinear geometry is included, at a load of 9.7 k just before the shell fails. The external midspan moment due to this load is 80.6 inch-kips. An integration of the internal stresses at midspan gives a tensile force of 7.86 kip and a compression force of 7.97 kips. Taking into account the deformed geometry, the internal moment at midspan can be found by integrating longitudinal stresses times lever arms to be 80.4 inch-kips. Thus the statical check on the analytical results is excellent.

This example verifies that including the change in geometry in the analysis of the cylindrical shell using NASHL will give results which predict the increase in the ultimate load as found in the experimental test.

EXAMPLE 2 - GABLED HP SHELL

In this example, a gabled HP shell is analyzed to determine its response behavior under the influence of dead and live loads on the shell. The importance of the nonlinear geometry and of the creep and shrinkage effects are demonstrated.

The dimensions of the gabled HP shell used in the present example are shown in Fig. 13. The shell has plan dimensions of 80 by 80 ft, a rise of only 8 ft, and a constant thickness of 3 in. The size of the crown beams and the edge beams are, respectively, $B \times H$ and $b \times h$. A number of linear and nonlinear analyses have been carried out using NASHL, all with the same edge beam dimensions $b \times h = 12 \times 16$ in., but with two different sizes for the crown beam $B \times H = 24 \times 8$ in. and 48×12 in. Only the results for the larger crown beam cases ($B \times H = 48 \times 12$ in.) will be presented here to show the undesirable effect of using an oversized crown beam.

In this example, first, a linear analysis for the total dead load (DL) of the shell plus edge beams is carried out using NASHL and the results are compared with an analysis using the well known SAP computer program to verify the NASHL finite element model. Second, selected results from a number of nonlinear analyses using NASHL are presented to illustrate the dramatic decrease in the ultimate load, which can occur when nonlinear geometric, material and time dependent effects are included.

Because of symmetry, only one quarter of the shell had to be used in the analytical model. For the NASHL analyses either a 4×4 or 2×2 mesh element layout was used, while for the SAP analyses, which uses a lower order finite element, an 8×8 mesh was used for all analyses.

Linear Analysis

The linear analysis was performed on an uncracked homogenous concrete system in which $E_c = 3.0 \times 10^3$ ksi and $\nu = 0.15$. Results from NASHL (4×4 mesh) and SAP (8×8 mesh) for vertical displacements and bending moments along the crown beam and the edge beam (Figs. 15 and 16) and for the shell membrane forces and bending moments (Fig. 17) are compared for an application of the total dead load of the shell plus beams. The agreement of results from NASHL and SAP is quite good. The important influence of the crown beam weight on its displacement is particularly noted in Fig. 15, where the effects of dead load of crown beam, shell and edge beams, and totals are shown separately

Nonlinear Analysis

A number of different analyses were made with NASHL to determine the effect of various nonlinearities [A14]. Only selected results from four analyses are presented in Figs. 18 to

23. The reinforcement layout (Fig. 14) for the shell and beams is designed based upon membrane theory for dead load (DL) plus a live load (LL) of 20 psf.

The material properties used in the analysis for the concrete and reinforcing steel are:

<u>Concrete</u>	<u>Steel</u>
$E_o = 3.33 \times 10^3 \text{ ksi}$	$E_s = 29.0 \times 10^3 \text{ ksi}$
$f'_c = 3.0 \text{ ksi}$	$E_{sh} = 0.0$
$f'_t = 0.471 \text{ ksi}$	$f_y = 60.0 \text{ ksi}$
$\nu = 0.15$	$\epsilon_{su} = 0.1$
$\beta = 0.5$	
$\epsilon_c = 2f'_c/E_o$	

ACI recommendations for the creep and shrinkage effects are used for the time dependent analysis.

All nonlinear analyses are performed by first applying the total dead load of the shell plus beams and then adding multiples of the 20 psf live load until failure occurs.

Crown displacement results from the analysis of a homogenous uncracked concrete model without (linear) and with nonlinear geometry effects included (Fig. 18) indicates the ultimate load in the latter case is DL + 11.8 LL. The response is almost linear up to a load of DL + 10.0 LL after which the continuous change in geometry causes local instability in the vicinity of the crown, where the curvatures of the shell are smallest (Fig. 23) and the axial force is the largest.

Fig. 19 shows the results of three analyses for a reinforced concrete analytical model of the HP gable shell which included nonlinear materials effects such as cracking, etc. First, for a short time loading with only nonlinear materials, the ultimate load was DL + 6.5 LL. Second, for a short-time loading with nonlinear material and geometry, the ultimate load drops to DL + 3.3 LL. And third, for a case in which the dead load is first applied and then left on for five months during which time creep and shrinkage takes place resulting in increased deflections and redistribution of stresses and then increments of live load are applied on the structure with nonlinear material and geometry, the ultimate drops to DL + 1.5 LL. This illustrates the large decrease in the ultimate load that can occur in shells of this type as each nonlinearity is included in the analysis.

This reduction in load capacity is primarily due to the

redistribution of axial force from the shell to the crown beam and thereby an amplification of the $P - \Delta$ effect in the crown region. This redistribution occurs due to cracking and the time dependent effects of creep and shrinkage.

Figs. 20 and 21 indicate the large increase in crown beam displacements and the decrease in shell axial membrane forces, and thus larger axial forces in the crown beam which occur due to time dependent effects between the initial dead load application at 28 days and five months later at 180 days. It can also be noted that the results by NASHL using a 4×4 and 2×2 finite element mesh are quite similar.

The crack patterns at ultimate load for the three nonlinear material analyses (Fig. 19) are shown in Fig. 22. In all cases, cracking is initiated near the support and propagates upwards along the diagonal. The failure mode is ductile for the case of short time loading with only nonlinear material; however, when both nonlinear material and geometry are included, the modes of failure are brittle for both the short time and the sustained load on the shell. In the latter case, the additional cracking in the supporting members further deteriorates the shell stiffness and results in the ultimate collapse of the structure.

CONCLUSIONS

A brief summary of a large number of linear and nonlinear analysis computer programs for shells and folded plates developed at the University of California has been presented. The latest nonlinear program NASHL can be used to perform a nonlinear material, geometric and time dependent analysis of reinforced concrete shells with edge beams through their elastic, cracking, inelastic and ultimate ranges. It was found that significant differences in the calculated ultimate load occurs as each of the nonlinear factors is included in the computer analysis. Future designs of large span shells should take these factors into account.

ACKNOWLEDGEMENT

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APPENDIX 1. COMPUTER PROGRAMS FOR THE ANALYSIS OF REINFORCED CONCRETE SHELLS

This Appendix 1 contains a list of the computer programs developed at the University of California, Berkeley since 1963 for the analysis of reinforced concrete shells. All of the programs, except Nos. 19, 22, 23 and 25 are based on the linear elastic analysis of uncracked homogeneous structures. Nos. 19, 22, 23, 25 (NARCS, NOTACS, NOPARC, NASHL) are nonlinear analysis programs which trace the response of the shell structures under various nonlinear effects.

For each computer program a short program name of four to seven letters is given followed by the title of the program. Figures in parentheses indicate the year the program was first used and figures in brackets indicate references in Appendix 2 where descriptions of the programs may be found.

Appendix 2A gives a sequential reference list of UC/SESM research reports which give detailed input-output information and numerical examples for the computer programs. Appendix 2B gives a sequential reference list of technical papers published in journals which give a brief description of the programs, their theoretical development and numerical examples.

Anyone interested in any of the programs listed should first read the detailed references indicated and then if they wish to obtain a copy of the program itself, they should write for further information to:

Professor A.C. Scordelis
Department of Civil Engineering
729 Davis Hall
UNIVERSITY OF CALIFORNIA
Berkeley, California 94720

BERKELEY COMPUTER PROGRAMS FOR CONCRETE SHELLS

1. FOLPLOR (1963) [A6,B2]
Analysis of Folded Plates by Ordinary Theory.
2. FOLPLEL (1963) [A6,B2]
Analysis of Folded Plates by Elasticity Theory.
3. MULEL (1963) [A6,B1,B7]
Analysis of Multiple Cylindrical Shells and Folded Plates with Arbitrary Edge Beams.
4. MULTPL (1965) [A1,B2,B3,B7,B8]
Analysis of Simply Supported Cellular Folded Plate Structures.

5. PRECYL (1965) [A6,B7]
Analysis of Simply Supported Prestressed Circular Cylindrical Shells Under Uniform Vertical and Dead Load.
6. MUPDI (1966) [A1,B4,B7,B8]
Analysis of Folded Plates Simply Supported at the Ends with Interior Rigid Diaphragms or Supports.
7. DOME (1968) [A6]
Analysis of Spherical Domes with or without Edge Ring Beams Under Axisymmetric Loading.
8. AXISB (1968) [A6]
Analysis of Axisymmetric Thin Shells with or without Edge Beams Under Axisymmetric Loads.
9. HYPARP (1968) [B5]
Analysis of Membrane Stresses in Hyperbolic Paraboloid Shells having a Parallelogram Shape in Plan.
10. HYPARQ (1968) [B6]
Analysis of Membrane Stresses in Hyperbolic Paraboloid Shells having an Arbitrary Quadrilateral Shape in Plan.
11. GROINV (1970) [B13]
Analysis of Membrane Stresses in Groined Hyperbolic Paraboloid Vaults.
12. CURSTR (1970) [A4,B8,B9]
Analysis of Curved Folded Plate Structures Using the Finite Strip Method.
13. FINPLA (1970) [A2,A3,B7,B8]
Analysis of Prismatic Folded Plates with Plate and Beam Elements.
14. CELL (1970) [A5,B8,B10]
Analysis of Cellular Structures of Arbitrary Plan Geometry.
15. MUPDI3 (1971) [A8,B4,B8]
Analysis of Folded Plates Simply Supported at the Ends with Interior Flexible Diaphragms or Planar Rigid Frame Support Bents.
16. FINPLA2 (1971) [A9,B8]
Analysis of Non-Prismatic Folded Plates with Plate and Beam Elements.
17. MULELR (1971) [A7]
Analysis of Multiple Cylindrical Shells and Folded Plates with Edge Beams and Under Uniform or Partial Loads.

18. MULDI (1971) [A7]
Analysis of Multiple Shell, Plate and Beam Element Systems Under Uniform or Partial Loads with Interior Rigid Diaphragms.
19. NARCS (1973) [A10,B11,B12]
Nonlinear Analysis of Reinforced Concrete Slabs and Shells.
20. CURDI (1974) [A11]
Linear Elastic Analysis of Curved Bridges on Flexible Bents.
21. DOMCYL (1975) [A6]
Analysis of Dome-Ring Beam-Cylinder Systems Under Axisymmetric Loading.
22. NOTACS (1976) [A12,B14]
Nonlinear Analysis of Reinforced Concrete Panels, Slabs and Shells for Time-Dependent Effects.
23. NOPARC (1979) [A13,B15]
Nonlinear Material, Geometric and Time-Dependent Analysis of Reinforced and Prestressed Concrete Slabs and Panels.
24. LSGNV (1979) [A6]
Analysis of Membrane Stresses in Hyperbolic Paraboloid, Elliptical Paraboloid and Cylindrical Groined Vaults.
25. NASHL (1982) [A14]
Nonlinear Analysis of Reinforced Concrete Shells with Edge Beams.
26. MUPDI4 (1985) [A15]
Elastic Analysis of Straight, Prismatic, Prestressed Cellular Folded Plate Structures with Optional Intermediate Diaphragms and Supports.
27. CURDI4 (1985) [A15]
Elastic Analysis of Circularly Curved, Prismatic, Prestressed Cellular Folded Plate Structures with Optional Intermediate Diaphragms and Supports.
28. CELL4 (1985) [A15]
Elastic Finite Element Analysis of Prestressed Cellular Structures of Arbitrary Plan Geometry and Constant Depth.

APPENDIX 2. REFERENCES ON COMPUTER PROGRAMS DEVELOPED AT
THE UNIVERSITY OF CALIFORNIA, BERKELEY, FOR
THE ANALYSIS OF REINFORCED CONCRETE SHELLS

A. RESEARCH REPORTS

1. Scordelis, A.C., "Analysis of Simply Supported Box Girder Bridges," Structural Engineering and Structural Mechanics Report No. SESM 66-17, University of California, Berkeley, October 1966.
2. Scordelis, A.C., "Analysis of Continuous Box Girder Bridges," Structural Engineering and Structural Mechanics Report No. SESM 67-25, University of California, Berkeley, November 1967.
3. Meyer, C., and Scordelis, A.C., "Computer Program for Prismatic Folded Plates with Plate and Beam Elements," Structural Engineering and Structural Mechanics Report No. SESM 70-3, University of California, Berkeley, February 1970.
4. Meyer, C., and Scordelis, A.C., "Analysis of Curved Folded Plate Structures," Structural Engineering and Structural Mechanics Report No. UC SESM 70-8, University of California, Berkeley, June 1970.
5. William, K.J., and Scordelis, A.C., "Computer Program for Cellular Structures of Arbitrary Plan Geometry," Structural Engineering and Structural Mechanics Report No. UC SESM 70-10, University of California, Berkeley, September 1970.
6. Scordelis, A.C., "Unpublished Class Notes for course CE 246, Design of Concrete Shells, Division of Structural Engineering and Structural Mechanics, University of California, Berkeley, January 1971.
7. Sethee, P.S., "Computer Solution of Multiple Shell and Plate Structures," M.S. Research Report No. 483, Division of Structural Engineering and Structural Mechanics, University of California, Berkeley, June 1971.
8. Lin, C.S., and Scordelis, A.C., "Computer Programs for Bridges on Flexible Bents," Structural Engineering and Structural Mechanics Report No. UC SESM 71-24, University of California, Berkeley, December 1971.
9. Meyer, C., and Scordelis, A.C., "Computer Program for Non-Prismatic Folded Plates with Plate and Beam Elements," Structural Engineering and Structural Mechanics Report No. UC SESM 71-23, University of California, Berkeley, December 1971.

10. Lin, C.S., "Nonlinear Analysis of Reinforced Concrete Slabs and Shells," Ph.D. Dissertation, Structural Engineering and Structural Mechanics Report No. UC SESM 73-7, University of California, Berkeley, April 1973.
11. Kabir, A.F., and Scordelis, A.C., "Computer Programs for Curved Bridges on Flexible Bents," Structural Engineering and Structural Mechanics Report No. UC SESM 74-10, University of California, Berkeley, September 1974.
12. Kabir, A.F., "Nonlinear Analysis of Reinforced Concrete Panels, Slabs and Shells for Time Dependent Effects," Ph.D. Dissertation, Structural Engineering and Structural Mechanics Report No. UC SESM 76-6, University of California, Berkeley, December 1976.
13. Van Greunen, J., "Nonlinear Geometric, Material and Time Dependent Analysis of Reinforced and Prestressed Concrete Slabs and Panels," Ph.D. Dissertation, Structural Engineering and Structural Mechanics Report No. UC SESM 79-3, University of California, Berkeley, October 1979.
14. Chan, E.C., "Nonlinear Geometric, Material and Time Dependent Analysis of Reinforced Concrete Shells with Edge Beams," Ph.D. Dissertation, Structural Engineering and Structural Mechanics Report No. UC SESM 82-8, University of California, Berkeley, December 1982.
15. Scordelis, A.C., Chan, E.C., Ketchum, M.A., Van der Walt, P., "Computer Programs for Prestressed Concrete Box Girder Bridges," Structural Engineering and Structural Mechanics Report No. UC SESM 85-02, University of California, Berkeley, March 1985.

B. TECHNICAL PAPERS

1. Scordelis, A.C., and Lo, K.S., "Computer Analysis of Cylindrical Shells," Journal of the American Concrete Institute, Vol. 61, No. 5, May 1964.
2. DeFries-Skene, A., and Scordelis, A.C., "Direct Stiffness Solution for Folded Plates," Journal of the Structural Division, Proceedings of American Society of Civil Engineers, Vol.90, No. ST-3, June 1964.
3. Scordelis, A.C., Davis, R.E. and Lo, K.S., "Load Distribution in Concrete Box Girder Bridges," ACI Proceedings of First International Symposium on Concrete Bridge Design, Toronto, Canada, April 1967, ACI Publication SP-23, 1969.
4. Scordelis, A.C., and Davis, R.E., "Stresses in Continuous Concrete Box Girder Bridges," ACI Proceedings of Second

International Symposium on Concrete Bridge Design, Chicago, April 1969, ACI Publication SP-26, 1971.

5. Ramirez, H.D., Scordelis, A.C., and Ngo, D., "Membrane Stresses in Hyperbolic Paraboloid Shells Having a Parallelogram Shape in Plan," Journal of the American Concrete Institute, Vol. 66, December 1969.
6. Scordelis, A.C., Ramirez, H.D., and Ngo, D., "Membrane Stresses in Hyperbolic Paraboloid Shells Having an Arbitrary Quadrilateral Shape in Plan," Journal of the American Concrete Institute, Vol. 67, January 1970.
7. Scordelis, A.C., "Analysis of Cylindrical Shells and Folded Plates," Proceedings, ACI Symposium on Concrete Thin Shells, New York, April 1970, ACI Publication SP-28, 1971.
8. Scordelis, A.C., "Analytical Solutions for Box Girder Bridges," Proceedings, Conference on Modern Developments in Bridge Design and Construction, Cardiff, Great Britain, April, 1971.
9. Meyer, C. and Scordelis, A.C., "Analysis of Curved Folded Plate Structures," Journal of the Structural Division, ASCE, Vol. 98, No. ST-1, January 1972.
10. William, K.J., and Scordelis, A.C., "Cellular Structures of Arbitrary Plan Geometry," Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol. 98, No. ST7, July 1972.
11. Lin, C.S., and Scordelis, A.C., "Nonlinear analysis of RC Shells of General Form," Journal of the Structural Division, Proceedings ASCE, Vol. 101, No. ST3, March 1975.
12. Lin, C.S., and Scordelis, A.C., "Finite Element Study of Reinforced Concrete Cylindrical Shell through Elastic, Cracking and Ultimate Ranges," Journal of the American Concrete Institute, Vol. 72, No. 11, November 1975.
13. Scordelis, A.C., "Analysis and Design of H.P. Groined Vaults," Proceedings, IASS World Congress on Space Enclosures, Montreal, Canada, July 4-9, 1976.
14. Kabir, A.F., and Scordelis, A.C., "Analysis of RC Shells for Time Dependent Effects," Bulletin of the International Association for Shell and Spatial Structures, Madrid, No. 69, April 1979.
15. Van Greunen, J., and Scordelis, A.C., "Nonlinear Analysis of Prestressed Concrete Slabs," Proceedings, Journal of Structural Engineering, ASCE, Vol. 109, No. 7, July 1983.

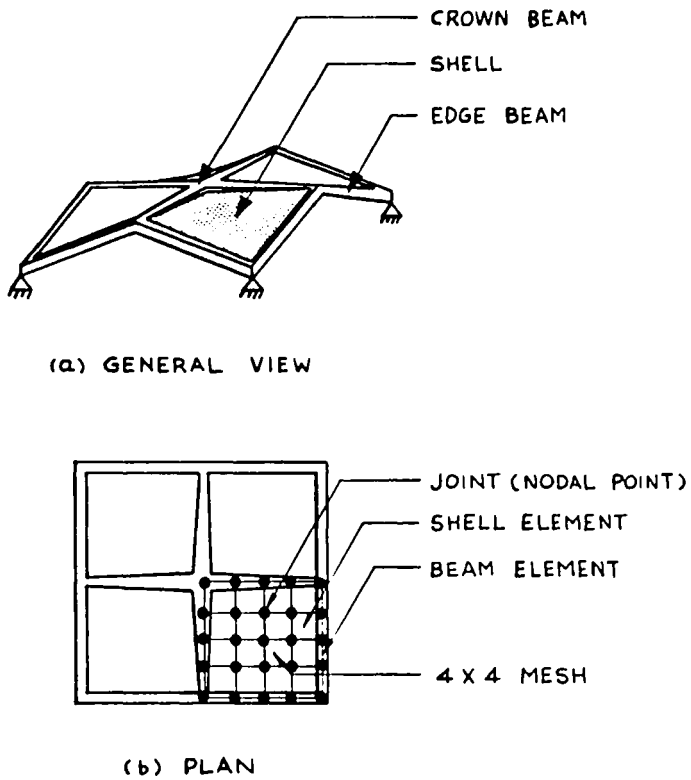


Fig. 1--Analytical model for NASHL computer program

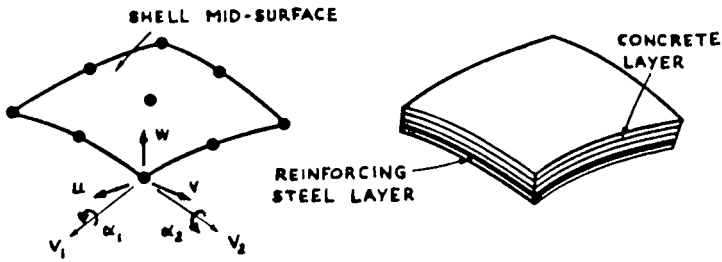


Fig. 2--Layered isoparametric RC shell element

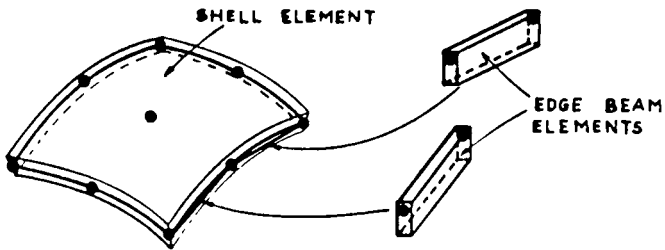


Fig. 3--Addition of two beam elements to curved shell element

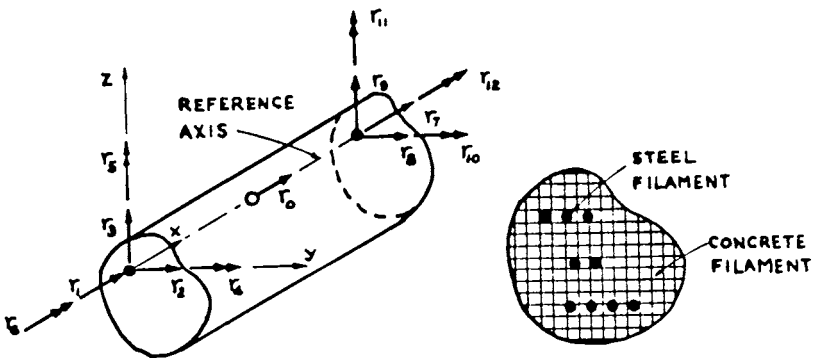


Fig. 4--Filament RC beam element

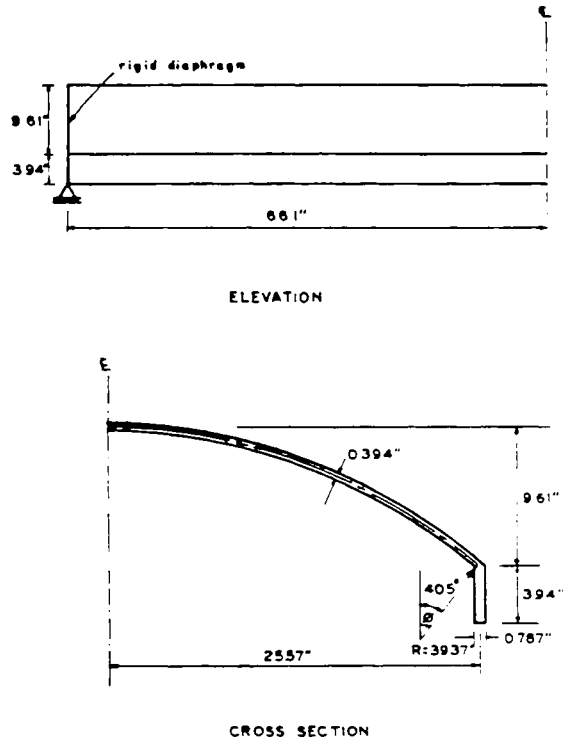


Fig. 5--(Example 1) Cylindrical shell geometry

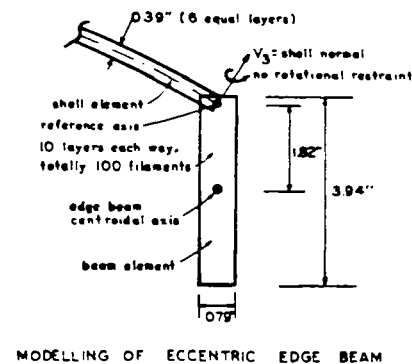
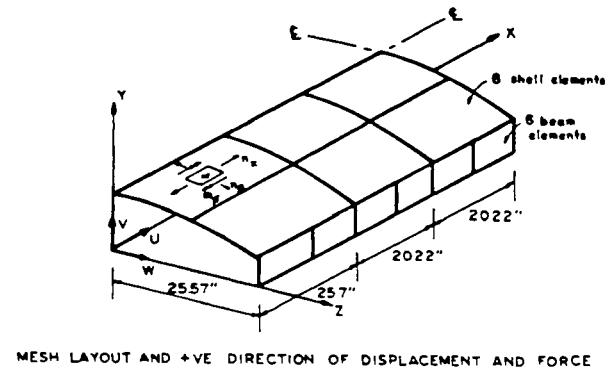
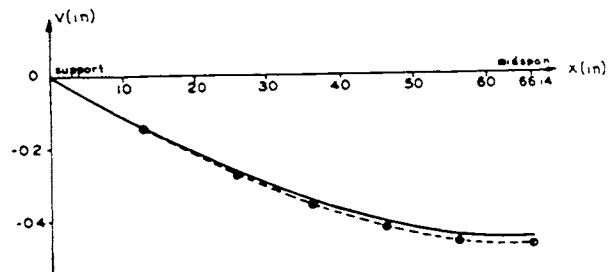
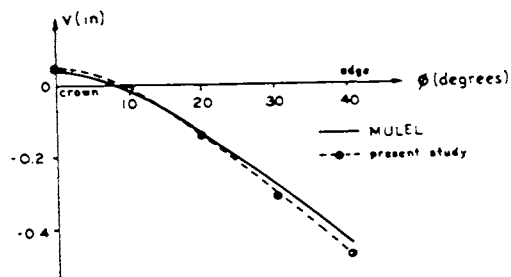


Fig. 6--(Example 1) Shell and beam mesh layout

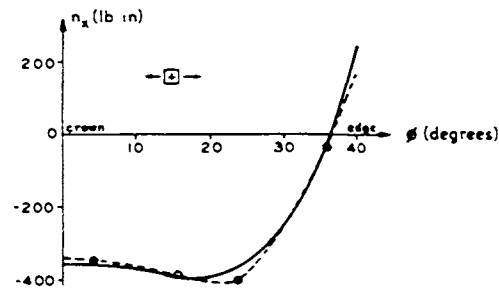


VERTICAL DISPLACEMENT V ALONG EDGE BEAM

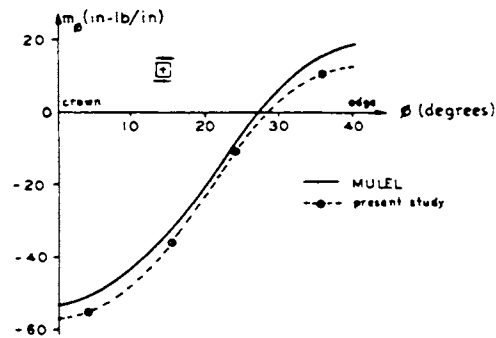


VERTICAL DISPLACEMENT V AT MIDSPAN

Fig. 7--(Example 1) Vertical displacements



SHELL MEMBRANE FORCE n_x AT MIDSPAN



SHELL TRANSVERSE BENDING MOMENT m_ϕ AT MIDSPAN

Fig. 8--(Example 1) Shell internal forces and moments

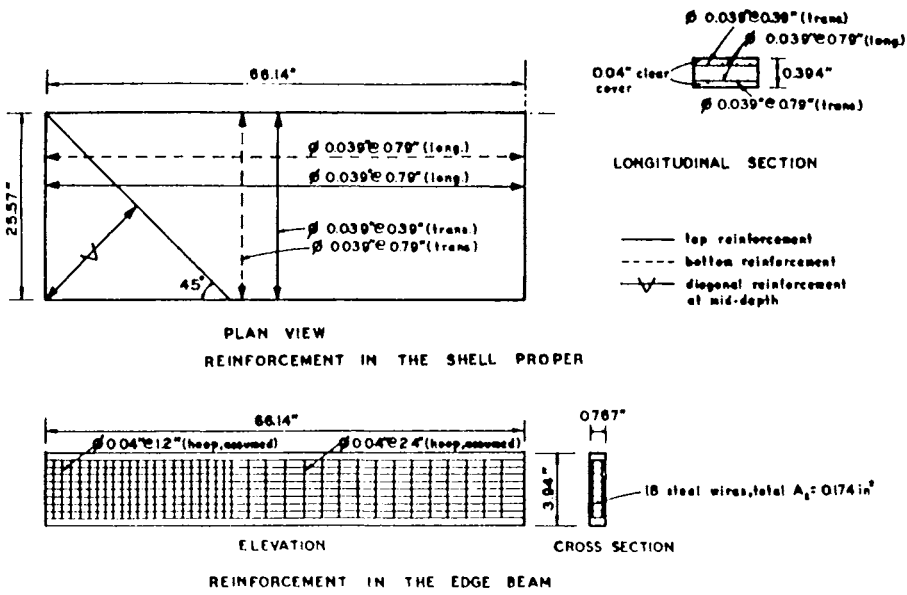


Fig. 9--(Example 1) Reinforcement for shell and beams

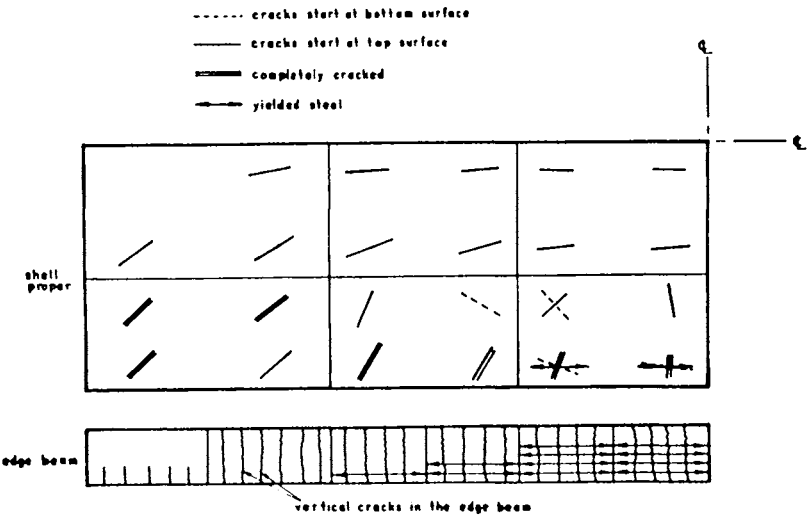


Fig. 10--(Example 1) Crack patterns at load of 10.2 kips